DIVISION OF STRUCTURES AND ENGINEERING SERVICES

TRANSPORTATION LABORATORY

RESEARCH REPORT

Evaluation of Compaction Grouting

FINAL REPORT

CA-DOT-TL-2163-1-76-61

OCT. 1976

76-61

Caltrans

CALIFORNIA DEPARTMENT OF TRANSPORTATION

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF STRUCTURES & ENGINEERING SERVICES
OFFICE OF TRANSPORTATION LABORATORY

October 1976

TL No. 642163

Mr. C. E. Forbes
Chief Engineer

Dear Sir:

I have approved and now submit for your information this final research project report titled:

EVALUATION OF COMPACTION GROUTING
07-Ora-57 PM 21.5/22

Study made by	Geotechnical Branch and District 07 Engineering Services
Under the Supervision of	Raymond A. Forsyth, P.E. Robert D. Allen, P.E.
Principal Investigators	Joseph B. Hannon, P.E. Leo D. Greiner, P.E.
Research Report by	Joseph B. Hannon, P.E. Wilfred S. Yee, P.E.

Very truly yours,

George A Hill

GEORGE A. HILL
Chief, Office of Transportation Laboratory

JBH/WSY:bjs
Attachment
Limited Distribution

TABLE OF CONTENTS

ACKNOWLEDGEMENTS

The work reported herein was part of a limited State-financed research project.

Appreciation is extended to the various District 07 Maintenance, Materials, and Surveys personnel who assisted with the field sampling, testing, monitoring and grouting program.

The contents of this report reflect the views of the Office of Transportation Laboratory which is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the State of California or the California Department of Transportation.

This report does not constitute a standard, specification, or regulation.

LIST OF TABLES

ACKNOWLEDGEMENTS

The work reported herein was part of a limited State-financed research project.

Appreciation is extended to the various District 07 Maintenance, Materials, and Surveys personnel who assisted with the field sampling, testing, monitoring and grouting program.

LIST OF TABLES

LIST OF FIGURES

LIST OF PHOTOGRAPHS

INTRODUCTION

Localized subsidence and long-term settlements are problems that plague many transportation engineers. These problems are sometimes unique to a particular area but various factors can contribute to the overall problem.

One localized area that has created a continuing maintenance problem is the long term settlement that has occurred at the south approach embankment of the Tonner Canyon Bridge, Road 07-Ora-57-21.5/22. This approach fill has been subject to repeated surface leveling treatments since its completion in 1971. Asphalt concrete up to a maximum thickness of 0.7 ft has been placed.

This problem was brought to the attention of the Transportation Laboratory in June 1974. It was believed that subsidence was occurring due to loose, unconsolidated foundation and embankment materials. After a review of the situation, it was decided to evaluate the possibility of alleviating the problem by means of localized embankment and foundation densification using compaction grouting.

Compaction grouting utilizes a very stiff mortar-like grout which compacts the soil by displacement. The technique consists of injecting a viscous cement based grout under pressure into a compactible soil mass to achieve controlled densification by physically moving the soil particles with bulbs of grout which act radially on the mass. Frequency of the "bulbs" center to center controls the degree of densification of the mass to be stabilized. The effectiveness of the technique will vary according to the soil type and density.

Compaction grouting has been used successfully by the private sector to stabilize building foundations, bridges, culverts and the soil under the tips of piles where settlements have occurred (1,2,3,4).

This report describes the equipment, materials, and procedures used in an experimental compaction grouting study at the Tonner Canyon Bridge approach. This study was performed to evaluate the use of compaction grouting to achieve localized densification of loose embankment or foundation materials to reduce the maintenance requirements associated with long-term settlement.

CONCLUSIONS

1. Loose fine grained soils can be densified by compaction grouting provided that proper techniques are applied and grout confinement can be achieved. In addition, high grouting pressures should be maintained to provide greatest densification using a stiff plastic grout mixture.

2. A water metering device is necessary to maintain control of grout consistency. High slump grout has a tendency to flow above the point of injection and prevents the withdrawal of equipment. Low slump grout provides greatest compactive effort.

3. It is important that all oversize aggregates be removed from the sand portion prior to incorporation in the mix. This prevents the difficulty encountered on this project due to pressure line rupture.

4. The grouting operation should be continuous, from start to finish. It was difficult to maintain a continuous operation during this study, since numerous problems developed.

5. With the exception of water control, the grout mix ingredients used in this study appeared adequate.

6. The 3-foot length of packer used in this study for grout injection was unsatisfactory and led to major difficulties. A 5-foot length would have probably resulted in a more efficient operation.

7. The most economical grouting operation would be accomplished in stages, starting at the bottom of the hole and advancing upward at 5 to 10 foot increments. Grouting should cease 10 to

15 feet below the surface to prevent heave. Pumping should continue at each stage until either refusal or surface heave is detected.

8. Attention should be given to lateral confinement during grouting. Grouting pressures should be reduced as lateral confinement diminished.

9. In analyzing the field data, it was difficult to define compactive beneficiation as a result of this preliminary grouting operation. Some densification was evidenced by higher sample densities, cone penetration measurements and drive sample resistance pressures.

10. Test data appear inconclusive for evaluating the adequacy of the 12-foot grid pattern. However, it is felt that a somewhat closer grid spacing may be necessary at this site. A streamlined operation with adequate equipment for monitoring mix ingredients, consistency, and grout delivery under higher pressures would verify the proper spacing. All future grouting holes on this project should be founded to a depth below 50 feet.

RECOMMENDATIONS

1. Since this study was unable to conclusively define improved soil densification as a result of compaction grouting at this site, it is suggested that either additional trial grouting be performed or consideration be given to a reassessment as to the need for this type of corrective treatment in lieu of future AC blanket repairs at this site.

2. It is recommended that supplemental sites be selected for conducting additional research and investigational work to further evaluate the uses of compaction grouting.

IMPLEMENTATION

The results of this study can be used as a guide for further evaluation of the compaction grouting procedure and reassessment of the correction proposed for the Tonner Canyon approach embankment.

Compaction grouting is a technique that could be used by the Office of Maintenance to alleviate the problem of long-term localized settlements.

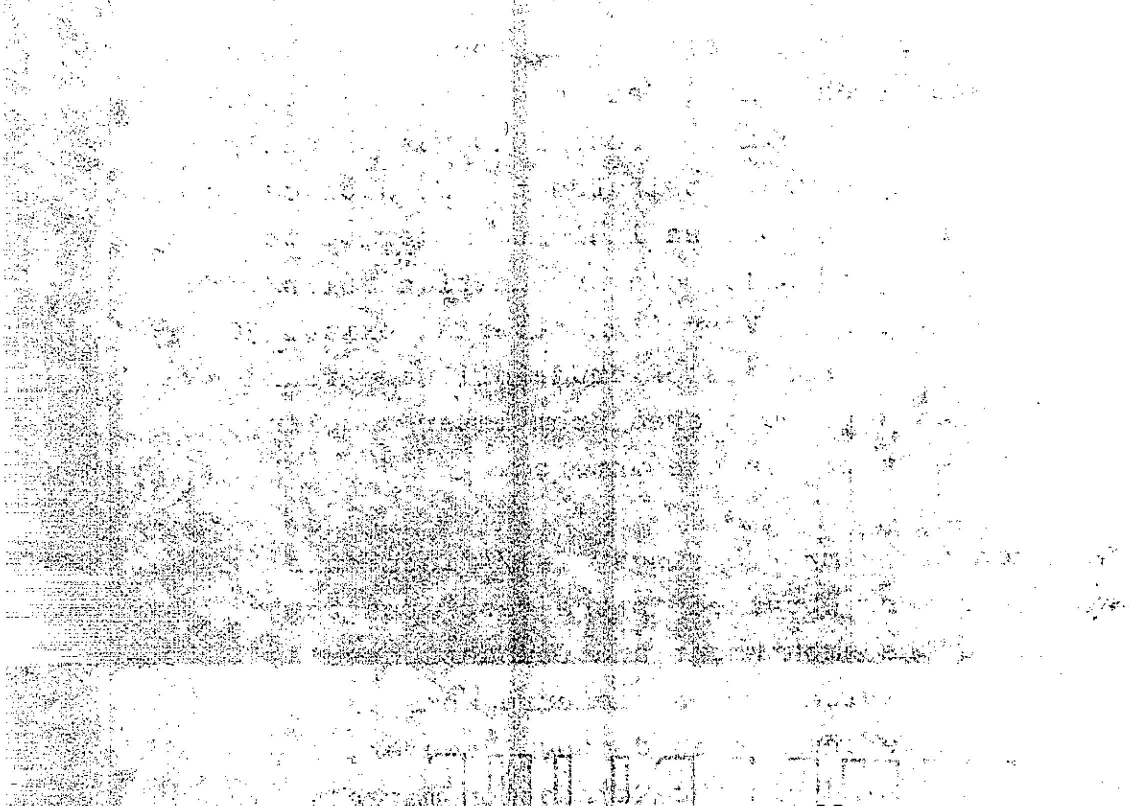
TEST SITE DESCRIPTION

The test site is located between Station 753 and 754 at the south approach to the Tonner Canyon Bridge, Road 07-Ora-57, PM 21.5/22. Figures 1, 2 and 3 show the plan and profile of the embankment area along with subsurface exploration data for the before test condition. This boring information was collected by District 07 in 1973 and 1974. The 30 foot high embankment fill at this location consists of a clayey silt sand with some gravel. The lower portion of the fill is made up of over-excavated and recompacted material as part of the design requirement, varying from a 10 foot thickness in the test area to a maximum thickness of 30 ft near Station 571+80. The adequacy of compaction of this material is questionable since this work was conducted in an area restricted by other earthwork operations. However, the Resident Engineer stated that inspection and testing indicated good compaction.

An oil field service road originally existed at about the lower limit of the over-excavated area. It is assumed that the "spoil" from this service road which was placed about 1920 was probably thrown down the slope and may have contributed to the subsidence problem. Borings taken in the test area indicate moderate to firmly compacted clayey silt and silty clay foundation soils with traces of oil and organic matter. Very firm material exists 50 to 60 ft below profile grade. Pockets of crude oil were found in boring 15 and oiley rags were recovered from later samplings of other borings. No free water was encountered.

In addition to the auger and sample borings, 2-1/4 inch cone penetrometer measurements were obtained in the test area to evaluate the looseness of the embankment and foundation soils for the before grouting condition. These measurements are variable and show no definite trend with depth. However, penetration resistance was higher in the foundation material. Refer to Figures 1 and 2.

A settlement platform under the bridge approach fill centerline at Station 755+30 indicated that 1.85 ft of settlement had occurred through July 1974. However, only 0.23 ft of this settlement developed since October 1970. The sag in the guardrail along the outside shoulder of the northbound lanes suggests that about 1-2 ft of settlement has occurred since construction (Photo 1). No distress is evident in the southbound lanes. A slope indicator was placed in July 1974 at Station 753+00 near the guardrail to a depth of 38 ft. Follow-up measurements made in February and April 1975 revealed that between 0.1 and 0.2-inch of movement had occurred towards the slope, down to the 34 ft. depth, possibly due to spreading of the fill. Subsequent measurements in December 1975 indicated that some additional movement had occurred.



The objective of a well planned grouting program is to insure densification of all soil intended to be improved. This requires proper layout of holes, grouting depths, and grouting sequence of holes. The depth of grouting must be sufficient to reach competent material, otherwise only part of the soil subject to settlement is being treated and an additional load is being imposed upon the underlying problem soil(4).

Grout holes are generally laid out on a basic grid over the area to be improved with the rows offset to give a triangular pattern of roughly equidistant spacing about 5 to 15 feet. In practice, however, it may not be practical to maintain an exact grid pattern because of obstructions and non-uniformity of the soil.

For this study, the grid pattern consisted of three test holes which were rotary predrilled. The boring logs are shown in Figures 5, 6, and 7. The holes were 3-7/8 inches in diameter to depths of about 65 feet. As shown in Figure 8, the triangular pattern of the holes was 12 feet apart to conform with lane width and to reduce problems associated with lane closure. Test holes 1, 2, and 3 were grouted in succession from bottom up and no casings were used during the operation.

Equipment and Materials

A mobile drill rig was used for drilling, sampling and grouting. Sand was transported to the test site with a mobile sand bunker with a water tank (Photo 2). The grouting materials were mixed with a plaster mixer using a 7 cubic foot capacity agitator, manufactured by Royal Industries. A Thomson Pump was used to pump the grout and deliver it through the stem of the drill rig. Refer to Photos 3, 4 and 5.

Materials for the grout mixture consisted of sand from Owl Rock Company, diatomite earth, cement and water. Gradings for the sand and diatomite earth are shown below.

GRAIN SIZE DISTRIBUTION
(Percentage Passing)

Sieve Size	Sand	Diatomite Earth
No. 4	100	100
No. 8	90	99
No. 16	77	97
No. 30	53	94
No. 50	22	90
No. 100	5	78
No. 200	1	68

Portland cement used in the grout mixture was Type I & II low alkali.

Experience by others suggests that the gradation and consistency of the sand used in the grout has a considerable influence upon the injection behavior and effectiveness of the grout. The objective is to provide a grout capable of remaining stable under pressures of 500-600 psi without excessive water loss. A stiff pumpable mix with nearly zero slump is generally recommended.

For this study, the grout mix consisted of the following proportions after several trial mixes:

Cement	30 lbs.	=	0.156 ft^3
Sand	420 lbs.	=	2.58 $ft.^3$
Earth	50 lbs.	=	0.50 $ft.^3$
Water	33 lbs.	=	0.53 $ft.^3$
Air (3%)	- -	=	0.12 $ft.^3$
	533 lbs.		3.89 ft^3/batch

A plot of the combined "as used" grading for the sand and diatomite earth is shown in Figure 9 which presents the preferred gradation limits reported by Warner and Brown(4) for a fine sand aggregate. The diatomite was included in the mix to improve pumpability.

A Homs Beam scale with a 105 lb capacity was used to proportion the materials for the grout mixture. Air content was not tested but was assumed to be three percent.

Pumping Rate and Pressure

Recommendations from previous investigations(1) and (4) indicate that the pump should be capable of displacing zero slump grouts with pressures up to 600 psi. The pump used for this study was capable of pressures up to 1,000 psi. Grout consistency and aggregate grading play an important role in pumping pressure. It is also important that all oversize aggregates be removed from the sand portion prior to incorporation in the mix. Equipment difficulties developed during the initial phase of this study due to oversize aggregate.

The maximum pressure developed and the quantity of grout injected are functions of the pumping rate. The pumping rate is influenced by the soil type, degree of compaction, moisture content, depth of injection, confining soil and structural load pressure.

Preliminary Grouting Tests

The purpose of these tests were to determine the maximum amount of grout that could be injected into a given test hole and the shape it would assume under the maximum required pressure. It was also equally important to determine the best consistency for the grout, the most effective method of grouting, and the amount of soil affected by one injection point. This information would provide the optimum grid spacing for grout holes on a future grouting contract to stabilize the entire approach fill on this project.

Initial grouting began on August 30, 1974 using Maintenance mud-jacking equipment but this operation was unsuccessful. The operation moved down station before the grouting could resume with different equipment at a later date. The new grid pattern is that which is presented on Figure 8. The operation resumed in April 1975, using the equipment described above.

Grouting for test hole No. 1 (Boring R-201, Figure 5) began from the bottom up at a depth of about 55 feet below the pavement surface. An air packer 1-1/4 inch inside diameter (Photo 6) was lowered into the hole and expanded under pressure to conform with and seal the hole at a depth of 55 feet. A 3 inch diameter hose was used to pump the grout from the mixing hopper through the drill rod stem to the point of application below the packer which was 3 feet in length. The grout slump was estimated at 2 inches. The grout was pumped until refusal was achieved. No pretreatment of the hole was performed before grouting.

The average time to deliver the first three batches was four minutes with pressures of 300 to 400 psi. A 23 minute delay was encountered following the third batch when the packer ruptured. A new packer was installed, however, the test hole refused to accept anymore fresh grout at the previous depth. The packer was then raised to a depth of 20 ft below the pavement surface. Two additional batches of grout were pumped into the hole averaging 3 minutes per batch at pressures of 150 psi. The packer was then raised to the 15 foot depth and five additional batches were pumped averaging 5 minutes per batch at 175 psi pressure until refusal was achieved. Pavement surface elevations taken before and after grouting in the vicinity of the test hole showed no significant change that would indicate pavement heave. Total delivery time for the 10 batches was 3-1/2 hours.

Test Hole No. 2 (Boring R-202, Figure 6) required two hours to complete the compaction grouting operation because the packer ruptured during inflation. A total of 6 batches equal to 22.8 ft^3 was forced into the hole. The packer was set at a depth of 55 feet for the first two batches, 50 feet for the next two batches and 45 feet for the fifth batch. The sixth batch was set at 20 feet. Average grouting pressures were 150 psi.

Test Hole No. 3 (Boring R-200, Figure 7) required 1-1/2 hours to complete using the packer which was shortened to 2 feet (Photo 7). A total of nine batches were forced into the hole. The packer was set at a depth of 22 feet for the first three batches and grouting pressures of 100 to 300 psi were obtained. Each batch required about 3 minutes for delivery. The packer was reset at 16 foot depth for the next two batches which were pumped until refusal was reached. Pressures of 300 psi and total delivery time of 10 minutes was required. The packer was placed near the pavement surface for the last four batches and pumping continued until pavement heave was noticeable. Pressures of 150 to 300 psi were required. Total delivery time for the last four batches was 30 minutes.

The 3 ft length of packer proved to be of inadequate length to seal the hole and prevent the grout from overflowing above the packer. Overflow can cause difficulty in packer recovery if the pressure grouting is delayed during pumping.

Grout consistency was also difficult to control without a water metering device.

Construction Control Monitoring

An elevation grid survey spaced at 5 ft intervals was established on the pavement surface in the vicinity of the test area. Heave stakes were also placed on an extension of the same grid which was expanded to a 10 ft spacing down the fill slope. Elevation measurements were taken on the pavement surface for control points before and after the grouting of each test hole. Heave stake elevations were recorded only before and after the complete trial grouting operation. These measurements were used to determine the grout intake and volumetric displacement of the embankment materials. A string line was used to monitor pavement heave during grouting (Photo 8).

EVALUATION

To evaluate compaction grouting, 2-1/4-inch Cone Penetrometer measurements were taken near the shoulder adjacent to the grouted holes subsequent to the grouting operation (Figure 8) and are identified as borings P-1 through P-6. They are also presented on Figures 10, 11 and 12. These measurements were compared with penetrometer readings taken before grouting which are identified by penetration borings B3C through B7C shown on Figure 2. The comparison of these penetrometer readings proved inconclusive in indicating an improvement in soil densification. However, the measurements before grouting were obtained prior to planning this study and were located about 10 feet closer to centerline than the boring in the actual test area. The follow-up data were more representative of the shoulder area. This information proved a poor comparison because a greater degree of confinement was available nearer the center of the embankment. It was assumed initially that these measurements could provide a before condition for relatively density.

Soil densities representative of the after condition were obtained in borings R-203, R-204 and R-205 (Figures 13, 14 and 15) which are located within or near the test grid pattern. A comparison of these densities with those representing the before condition can be made by inspection of Table 1. A review of these data again indicates the difficulty in making a valid comparison to establish improvement in density. However, if boring 15 is considered representative of the before condition, since it is located nearest to the test area, some density improvement is indicated.

When the soil densities in boring R-203 are compared with those in boring R-204, an average increase of about 1 lb/ft^3 is suggested for boring R-204 over boring R-203. Both borings are located within the test grid. Boring R-203 is located 3.8 ft from grout test hole 2 (boring R-202) and boring R-204 is 5.5 ft out from grout test hole 2 and possibly within the influence of all three grout test holes (refer to Figure 8). No before density measurements were obtained for the grout test holes (borings R-200, R-201 and R-202).

Since comparative measurements of density were not available for the before condition in the grout test holes, an attempt was made to relate the drive pressures required with the California Modified 2-inch sampler to increased soil density. This comparison is shown in Table 2. The average drive pressure ($\overline{x}$) for the full depth suggests an increase in sampling effort needed for the after grouting condition when borings R-202, R-203 and R-204 are compared. It is interesting to note that higher follow-up pressures were required at the 20, 45, 50 and 55 ft depths at which grout was pumped in test hole 2 which is an indication of densification.

Elevation profiles obtained on the pavement surface before and after the grouting operation suggest no significant net increase in surface elevation in the vicinity of either test hole 1 or test hole 2 as a result of grouting. However, surface elevations increased an average of 0.01 ft. in the vicinity of test hole 3. Some elevation points heaved as much as 0.10 ft. Total surface heave of the pavement represented slightly over 10 $ft.^3$ of displacement. The surface heave of test hole 3 was the result of pumping too near the pavement surface. No significant elevation changes were recorded on the fill slope heave stakes that could be related to the grouting operation.

As a result of pumping, a net total of 32.6 $ft.^3$ of grout displacement and compacted the soil in test hole 1. The grout take of test hole 2 was 16.5 $ft.^3$ and test hole 3 took 19.5 $ft.^3$.

Data collected from this field study proved insufficient for a conclusive evaluation of a trial grid pattern for compaction grouting at this site.

REFERENCES

1. Brown, Douglas R. and Warner, James, "Compacting Grouting", Journal of the Soil Mechanics and Foundation Division, ASCE, August, 1973, Vol. 99, No. SM8, Proc. Paper 9908, pp 589-601.

2. Brown, Douglas R. and Warner, James, "Compaction Grouting", Closure, Journal of the Geotechnical Engineering Division, ASCE, December 1974, Vol. 100, No. GT12, pp. 1300-1303.

3. Graf, Edward D., "Compaction Grouting Technique and Observation", Journal of the Soil Mechanics and Foundation Division, ASCE, September, 1969, Vol. 95, No. SM5, pp 1151-1158.

4. Warner, James and Brown, Douglas., "Planning and Performing Compaction Grouting", Journal of the Geotechnical Engineering Division, ASCE, June 1974, Vol. 100, No. GT6, pp. 653-666.

TABLE 1

DENSITIES BEFORE AND AFTER GROUTING

	BEFORE (lb/ft^3)					AFTER (lb/ft^3)		
Sample Depth (ft.)	(09)* 61 ft Rt. ℄ Sta. 752+85 2/73	49 ft Rt. ℄ Sta. 752+93.1 2/15/74	(10) 69 ft Rt. ℄ Sta. 753+00 2/73	(13) 29 ft Rt. ℄ Sta. 753+15 2/73	(15) 35 ft Rt. ℄ Sta. 753+42 2/73	(R-203) 55 ft Rt. ℄ Sta. 753+26.8 4/14-4/17/75	(R-204) 55 ft Rt. ℄ Sta. 753+28.5 4/21/75	(R-205) 46 ft Rt. ℄ Sta. 753+28.5 2/22/75
5		**110(6') ***120(6')				104.7 118.9	115.0 124.8	115.8 127.7
10	125.4	117(7') 129(7')	128.8		125	110.1 124.6	109.6 124.4	112.5 126.1
15		98.9(13') 112.5(13')	124.2	125.4		107.9 117.2	109.8 120.8	115.1 125.3
20		113 124	124.2		125.5	113.5 128.0	116.0 131.4	112.7 126.7
25		99.9 112	124.0			117.6 130.8	118.7 134.2	115.7 128.0
30	129.8	118 129		114.7	126.1	109.7 123.1	112.5 123.8	111.4 124.2
35			127	128.8	112.9	103.3 121.5	100.8 120.5	102.6 117.9
40		117 130	126.8			115.1 127.7	116.2 126.0	112.8 119.2
45		92(48') 107(48')	127	124.0		117.5 127.7	115.5 124.8	116.6 125.3
50		96 119	131	103.0	113	121.7 131.1	122.1 133.9	122.4 129.0
55	118.5		118.7			109.8 125.2	108.5 122.6	112.3 128.8
60			119.2			118.9 129.5	118.6 130.5	119.5 131.7
65			108.1			91.0 114.1	92.6 116.2	83.9 110.5
70			95.8			86.3 112.3	91.2 116.4	92.0 116.7

*Boring Identification **Dry Density ***Wet Density

TABLE 2

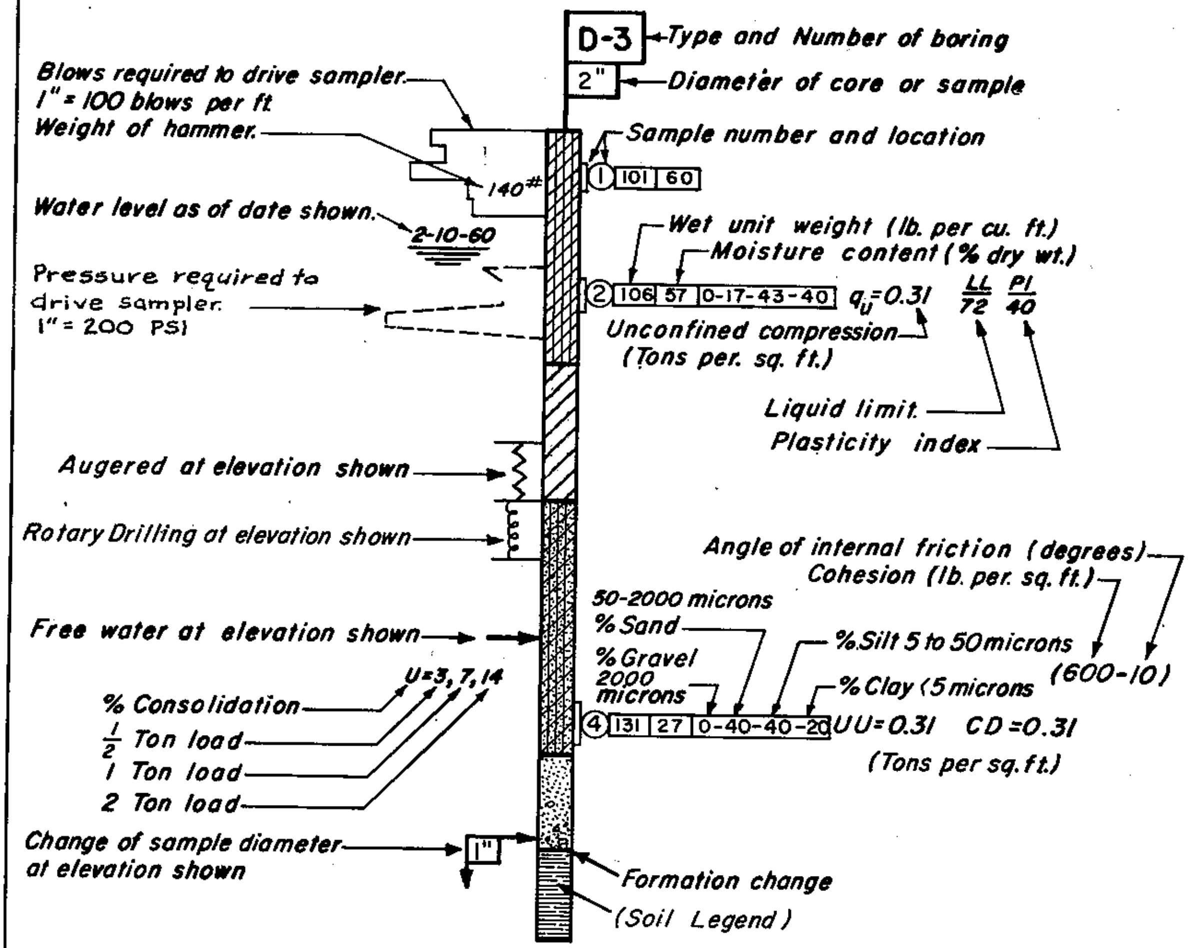
DRIVE PRESSURE (PSI) REQUIRED
WITH CALIF. MODIFIED 2-INCH SAMPLER

	BEFORE GROUTING			AFTER GROUTING		
Sample Depth (ft)	(R-200)* 61 ft Rt. ℄ Sta. 753+34	(R-201) 49 ft Rt. ℄ Sta. 753+34	(R-202) 55 ft Rt. ℄ Sta. 753+23	(R-203) 55 ft Rt. ℄ Sta. 753+26.8	(R-204) 55 ft Rt. ℄ Sta. 753+28.5	(R-205) 46 ft Rt. ℄ Sta. 753+28.5
5	150	400	-	250	350	450
10	250	250	200	150	300	200
15	150	200	250	300	300	300
20	175	350	400	400	400	300
25	300	350	300	350	450	300
30	275	200	300	350	200	300
35	200	350	350	400	250	350
40	200	350	200	350	250	-
45	200	350	250	300	250	250
50	-	350	250	400	250	250
55	300	300	250	450	200	400
N =	10	11	10	11	11	10
$\overline{X}$ =	220	314	275	336	291	310

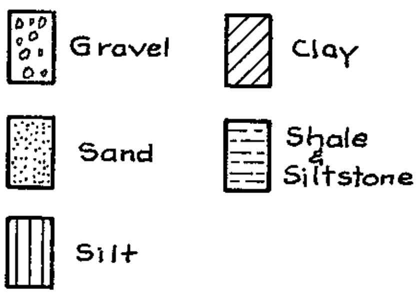
*Boring Identification

Fig. 4

BORING LEGEND
CROSS-SECTION & PROFILE SHEETS



SOIL LEGEND

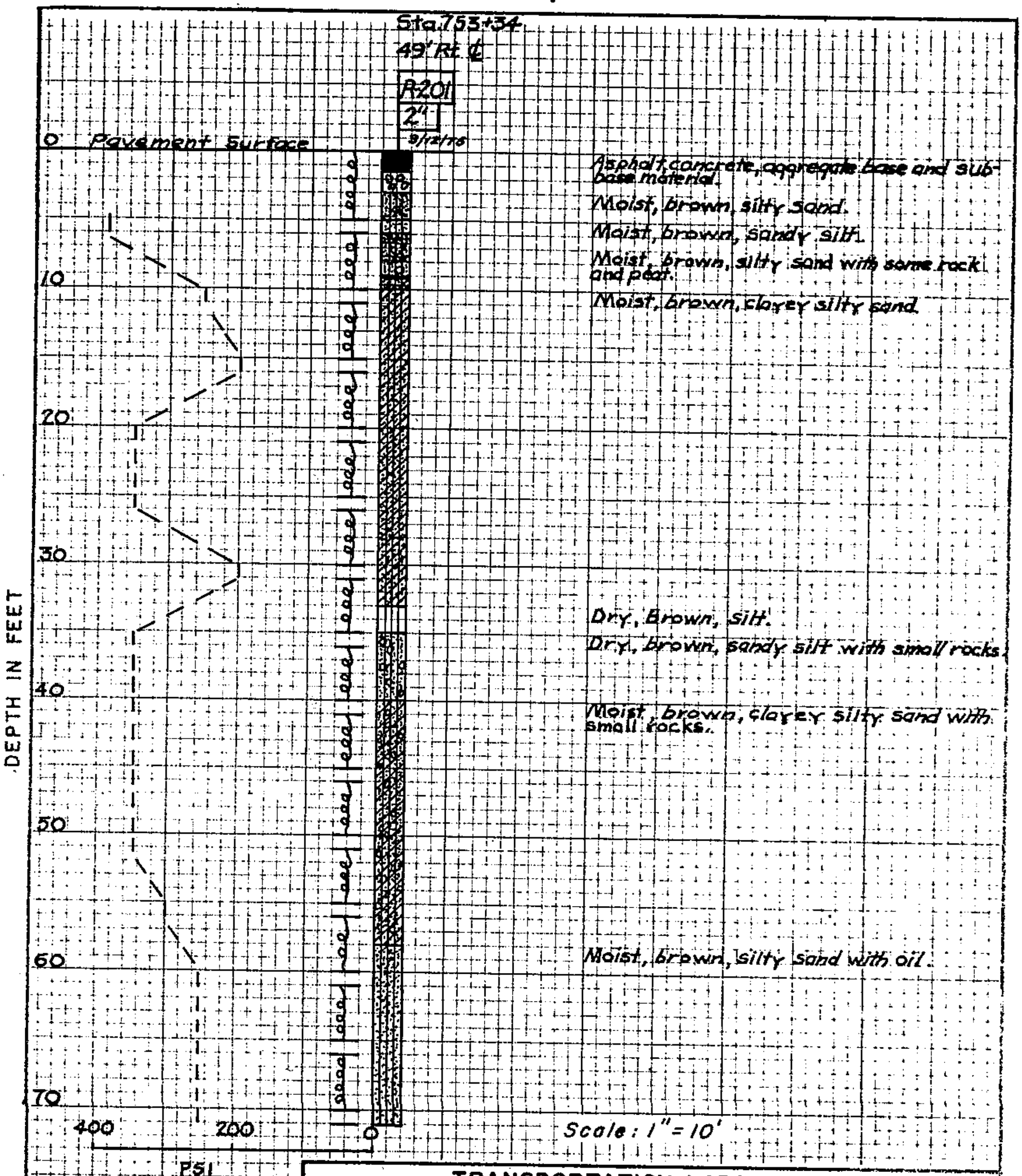


STRENGTH TESTS

q_u - Unconfined Compression
UU - Unconsolidated Undrained
CU - Consolidated Undrained
CD - Consolidated Drained

STATE OF CALIFORNIA HIGHWAY TRANSPORTATION AGENCY DEPARTMENT OF TRANSPORTATION TRANSPORTATION LABORATORY		
EVALUATION OF COMPACTION GROUTING - SOUTH APPROACH - TONNER CANYON BRIDGE		
DATE 6/76	SUBMITTED BY: Joseph Hannon SR. MAT'LS. & RES. ENG'R.	DWG. NO. 2163
DR. BY / CK. BY	APPROVED BY: [signature] SUP'G. MATLS. & RES. ENG'R. - FDN	SHEET NO 4 OF 15 SHTS.

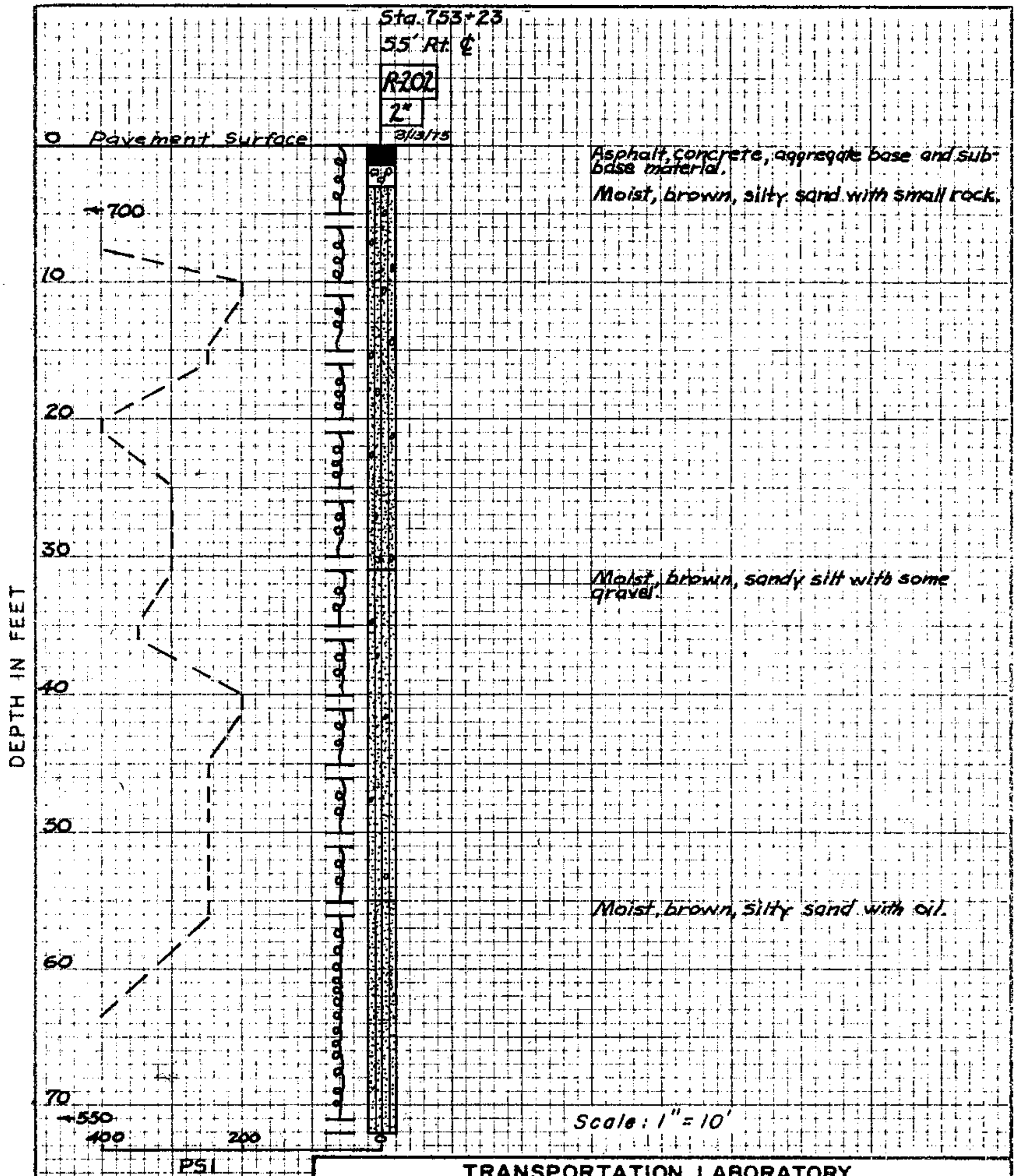
Fig. 5



TRANSPORTATION LABORATORY
FOUNDATION INVESTIGATION
EVALUATION OF COMPACTION GROUTING
SOUTH APPROACH-TONNER CANYON BRIDGE
Boring R-201

DATE	DIST.	COUNTY	ROUTE	POST MILE	DWG. NO.	TOTAL SHEETS
6/76	07	Ora	57	21.5/22	2163	5 of 15

Fig. 6



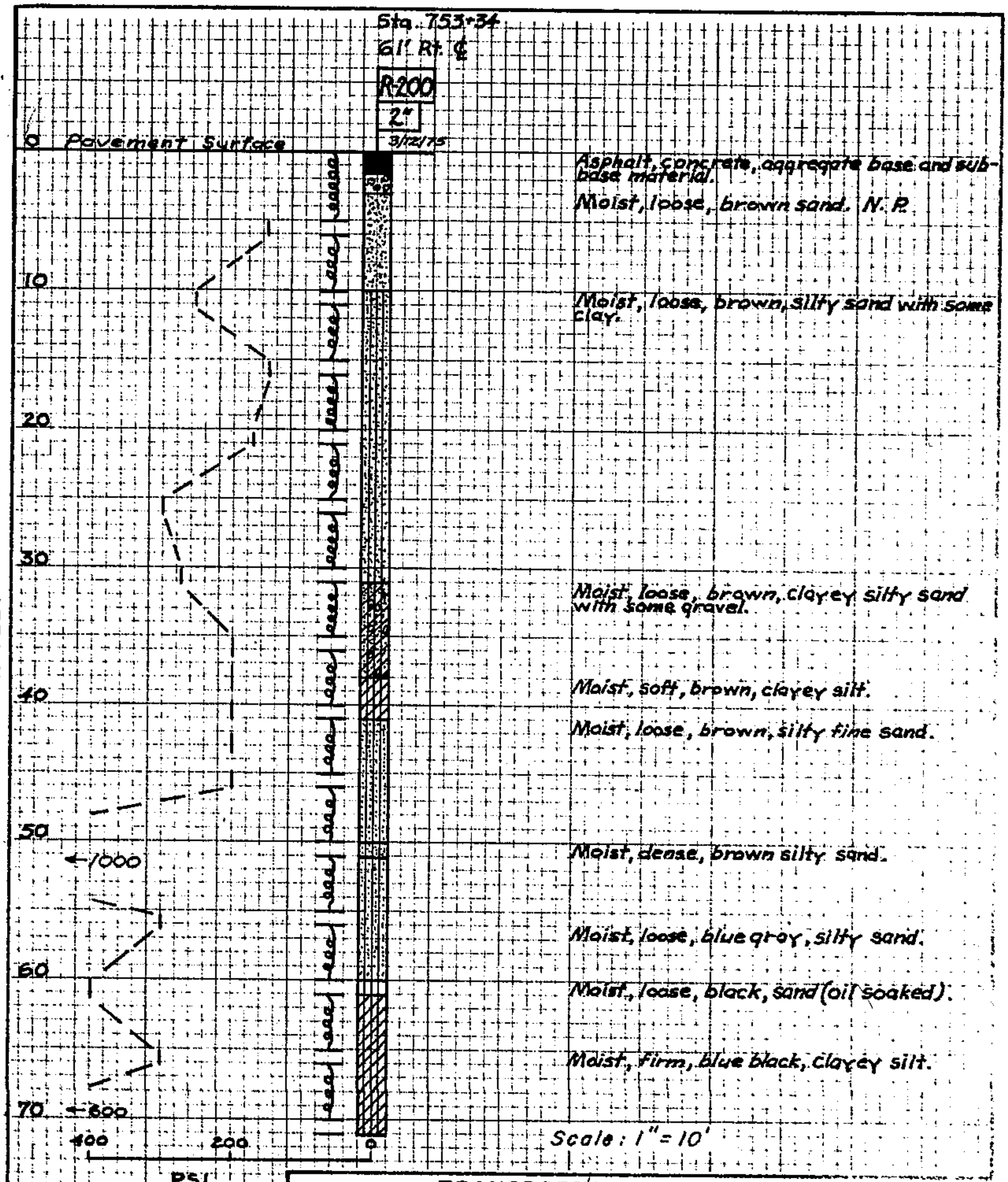
TRANSPORTATION LABORATORY

FOUNDATION INVESTIGATION

EVALUATION OF COMPACTION GROUTING
SOUTH APPROACH–TONNER CANYON BRIDGE
Boring R-202

DATE	DIST.	COUNTY	ROUTE	POST MILE	DWG. NO.	TOTAL SHEETS
6/76	07	Ora	57	21.5/22	2163	6 of 15

Fig. 7



TRANSPORTATION LABORATORY
FOUNDATION INVESTIGATION
EVALUATION OF COMPACTION GROUTING
SOUTH APPROACH-TONNER CANYON BRIDGE
Boring R-200

DATE	DIST.	COUNTY	ROUTE	POST MILE	DWG. NO.	TOTAL SHEETS
6/76	07	Ora	57	21.5/22	2163	7 of 15

Fig. 8

LAYOUT OF TEST GROUTING PATTERN AND SURFACE ELEVATION GRID

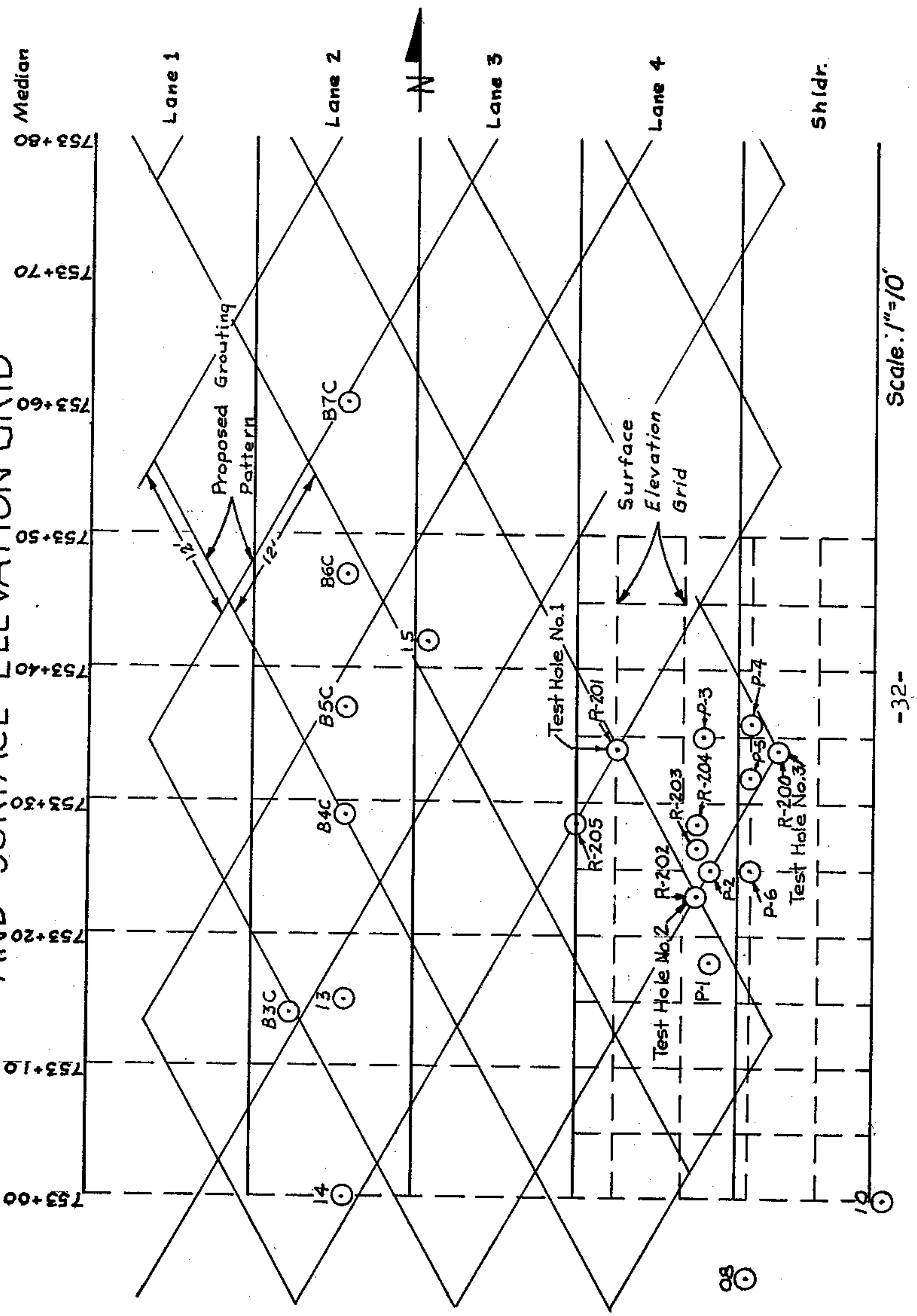
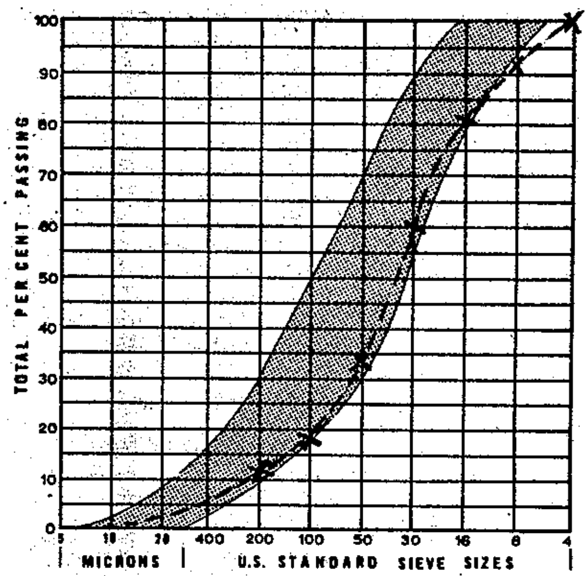


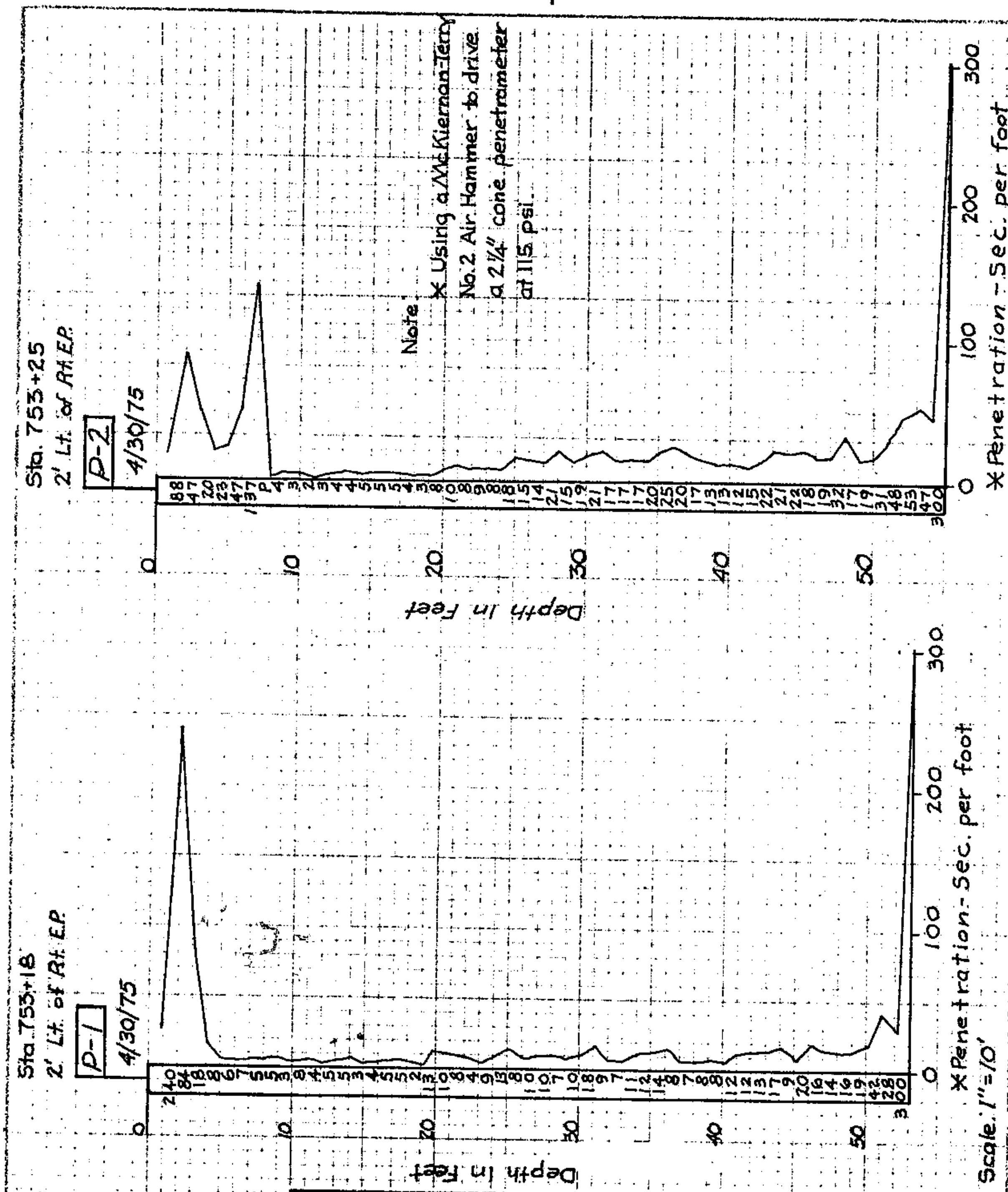
Figure 9



—Preferred Limits of Gradation for Sand Used for Compaction Grout

(from Warner and Brown (4))

Fig. 10



TRANSPORTATION LABORATORY

FOUNDATION INVESTIGATION

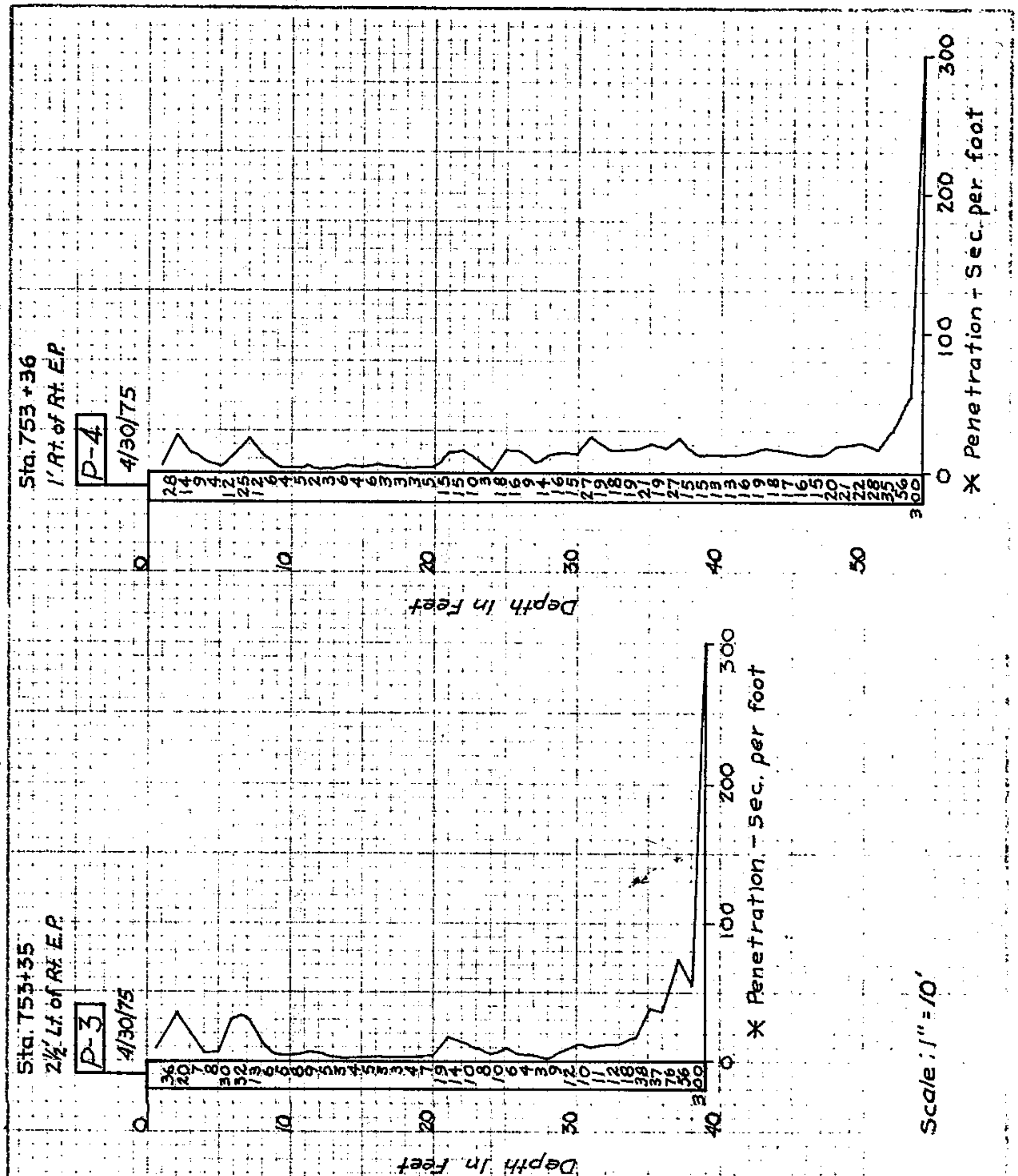
EVALUATION OF COMPACTION GROUTING

SOUTH APPROACH-TONNER CANYON BRIDGE

Penetration Borings P-1 and P-2

DATE	DIST.	COUNTY	ROUTE	POST MILE	DWG NO.	TOTAL SHEETS
6/76	07	Ora	57	21.5/22	2163	10 of 15

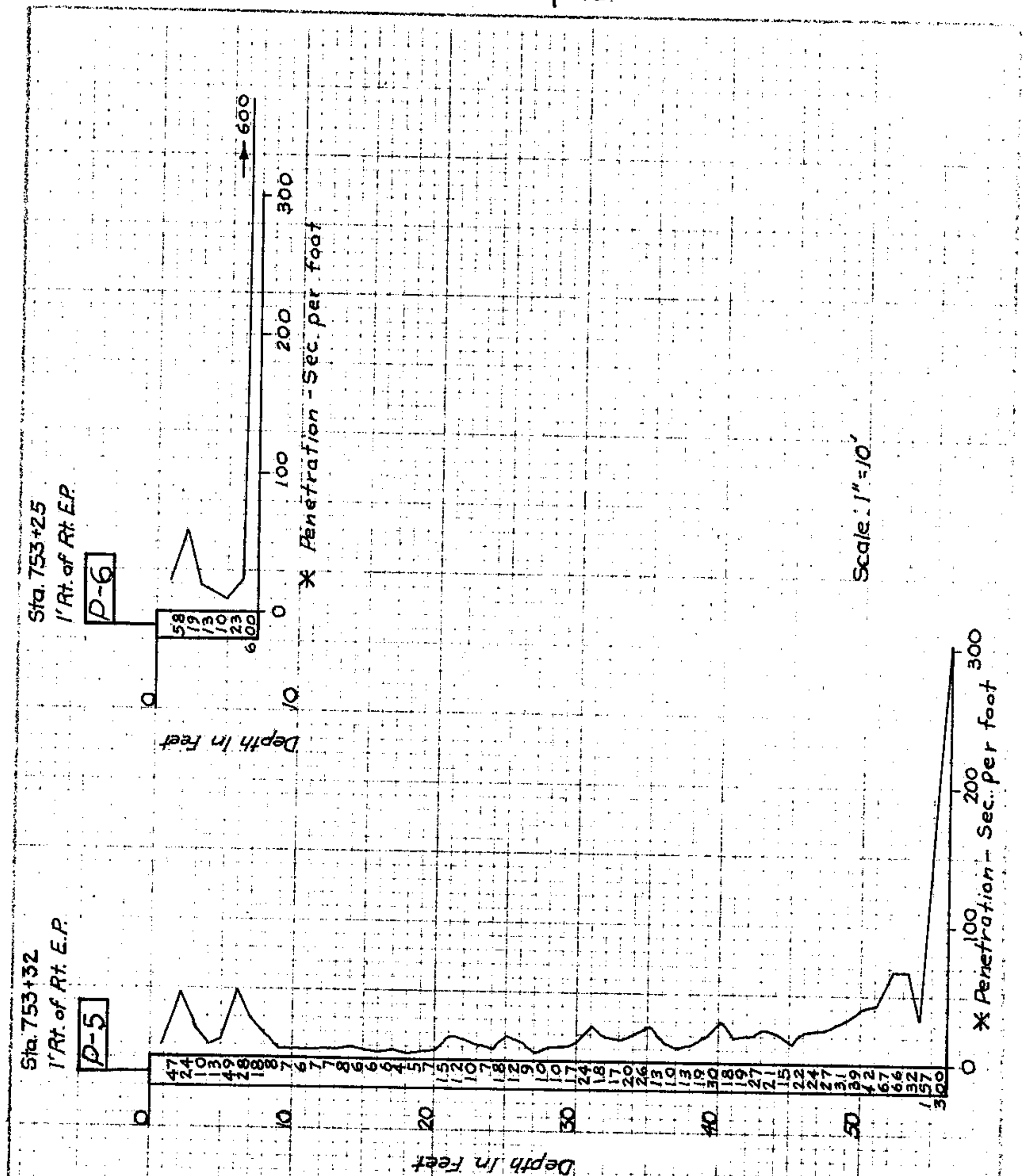
Fig. 11



TRANSPORTATION LABORATORY
FOUNDATION INVESTIGATION
EVALUATION OF COMPACTION GROUTING
SOUTH APPROACH - TONNER CANYON BRIDGE
Penetration Borings P-3 and P-4

DATE	DIST.	COUNTY	ROUTE	POST MILE	DWG. NO.	TOTAL SHEETS
6/76	07	Ora	57	21.5/22	2163	11 of 15

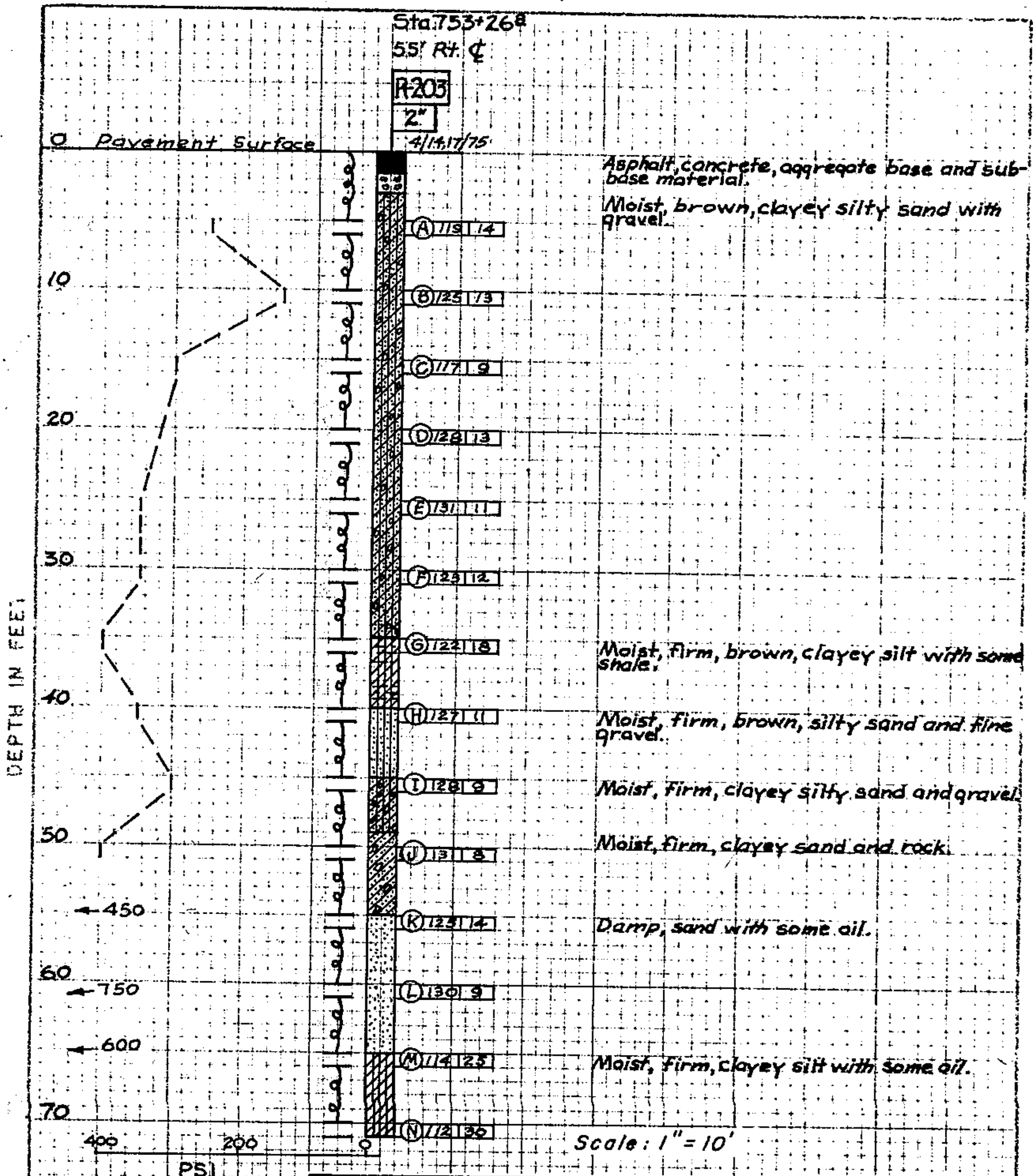
Fig. 12.



TRANSPORTATION LABORATORY
FOUNDATION INVESTIGATION
EVALUATION OF COMPACTION GROUTING
SOUTH APPROACH-TONNER CANYON BRIDGE
Penetration Borings P-5 and P-6

DATE	DIST.	COUNTY	ROUTE	POST MILE	DWG. NO.	TOTAL SHEETS
6/76	07	Ora	57	2.15/22	2163	12 of 15

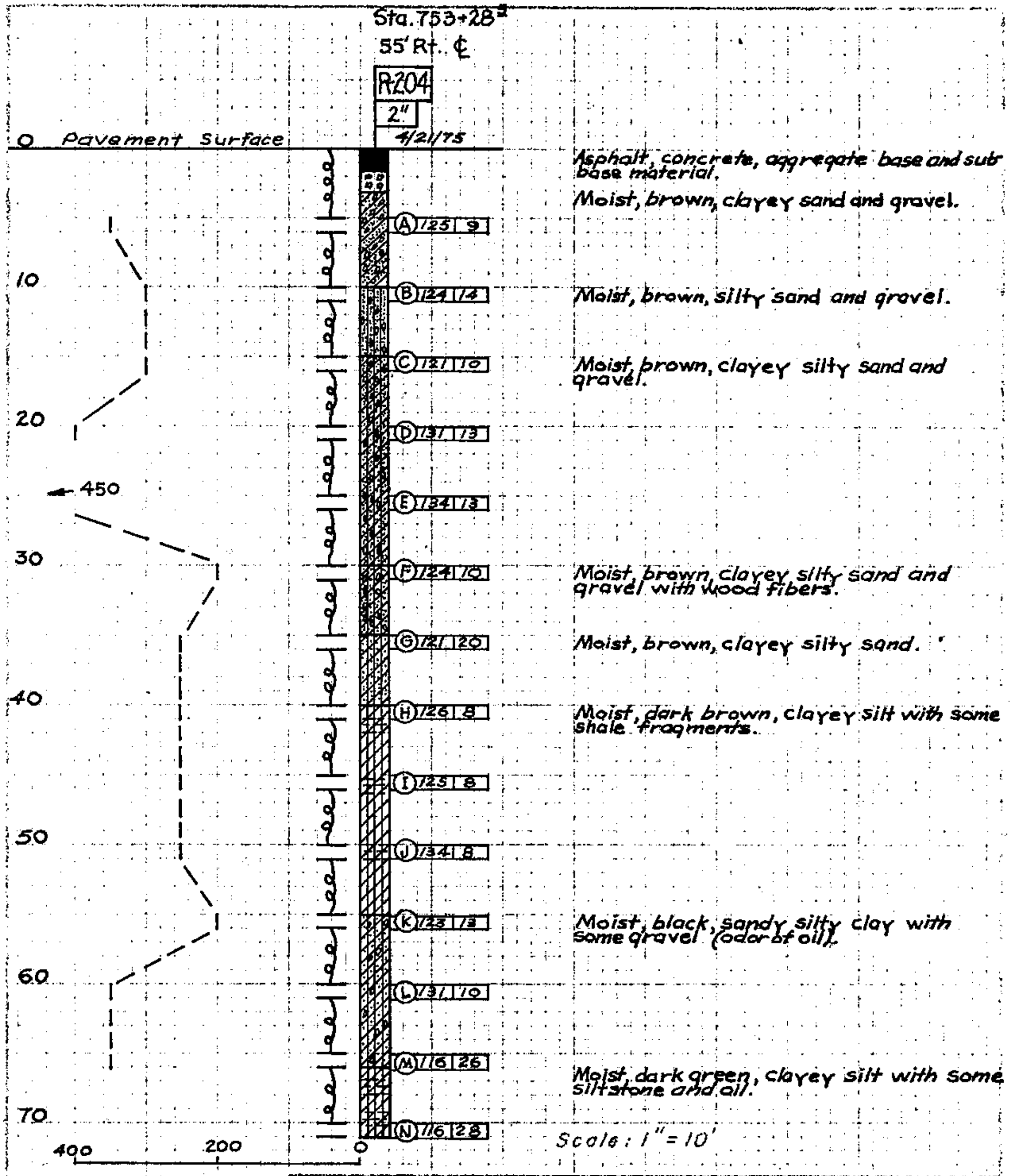
Fig. 13



TRANSPORTATION LABORATORY
FOUNDATION INVESTIGATION
EVALUATION OF COMPACTION GROUTING
SOUTH APPROACH-TONNER CANYON BRIDGE
Boring R-203

DATE	DIST.	COUNTY	ROUTE	POST MILE	DWG. NO.	TOTAL SHEETS
6/76	07	Ora	57	21.5/22	2163	13 of 15

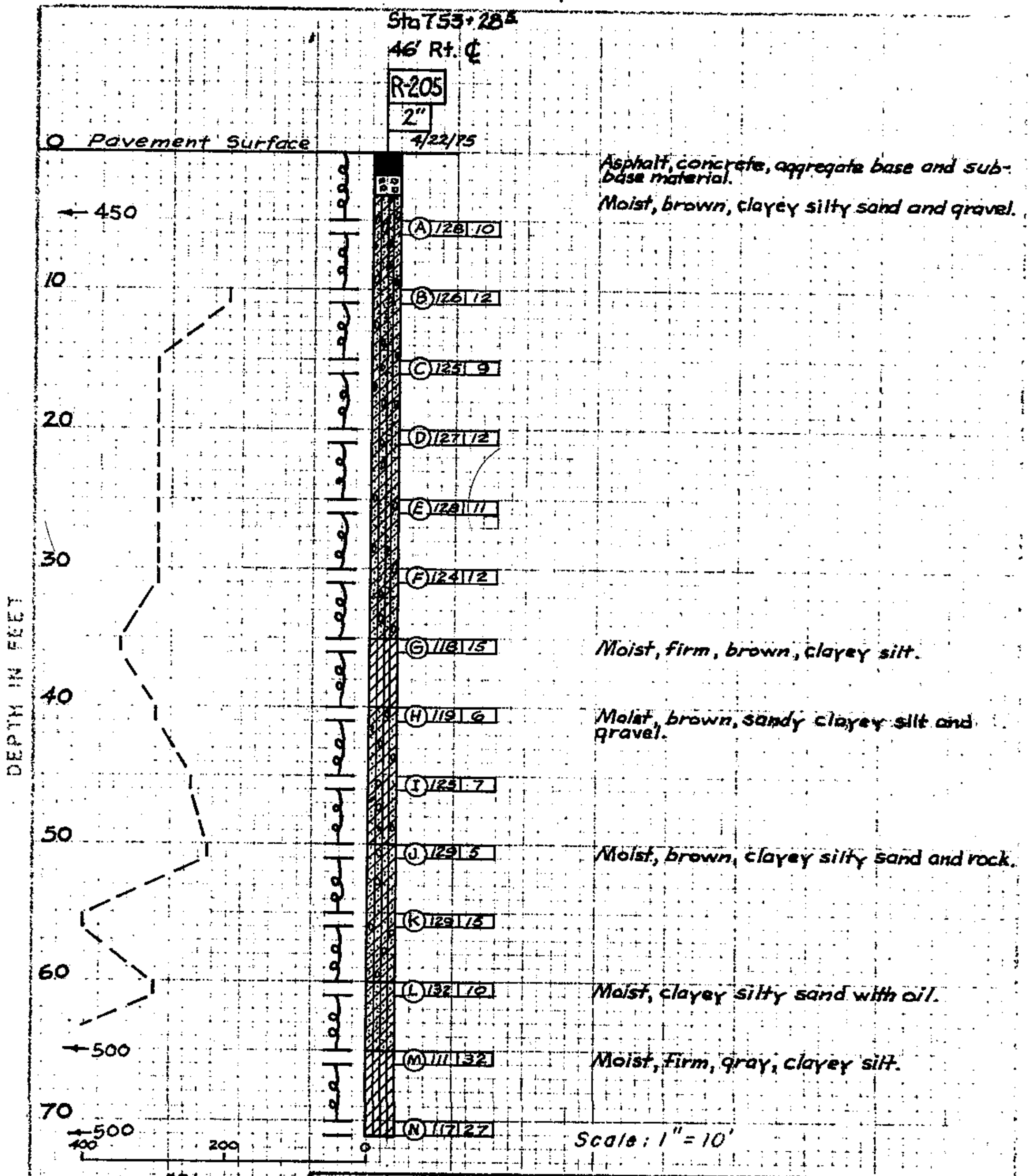
Fig. 14



TRANSPORTATION LABORATORY
FOUNDATION INVESTIGATION
EVALUATION OF COMPACTION GROUTING
SOUTH APPROACH-TONNER CANYON BRIDGE
Boring R-204

DATE	DIST.	COUNTY	ROUTE	POST MILE	DWG. NO.	TOTAL SHEETS
6/76	07	Ora	57	21.5/22	2163	14 of 15

Fig. 15



TRANSPORTATION LABORATORY
FOUNDATION INVESTIGATION
EVALUATION OF COMPACTION GROUTING
SOUTH APPROACH-TONNER CANYON BRIDGE
Boring R-205

DATE	DIST.	COUNTY	ROUTE	POST MILE	DWG. NO.	TOTAL SHEETS
6/76	07	Ora	57	21.5/22	2163	15 of 15

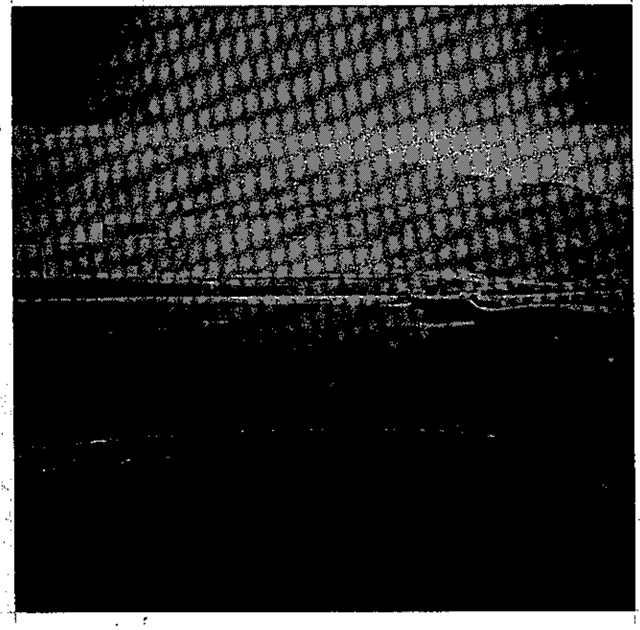


Photo 1: Sag in guardrail suggesting settlement of northbound lanes.

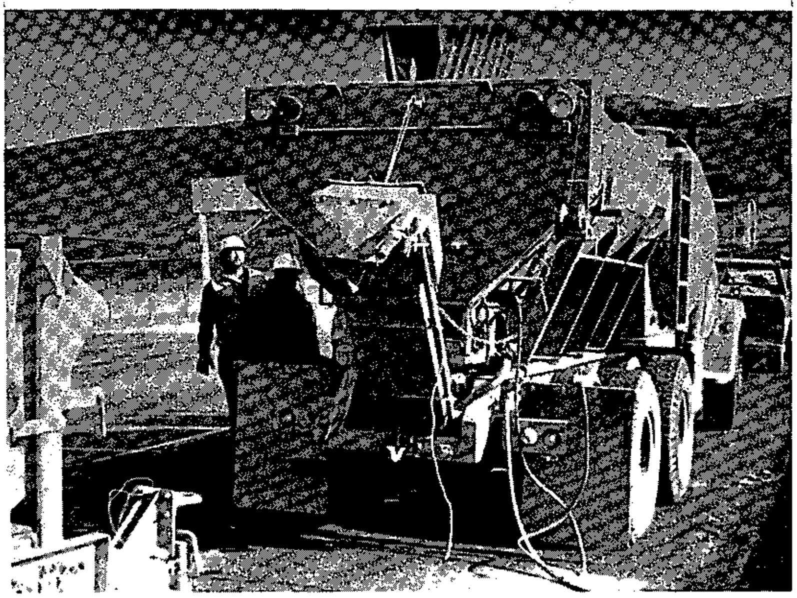


Photo 2: Mobile Sand Bunker with water tank.

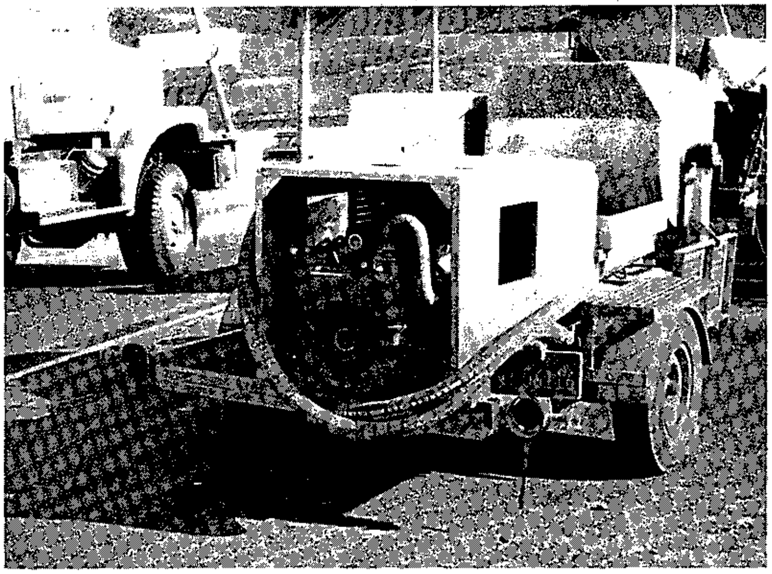


Photo 3: Plaster Mixer

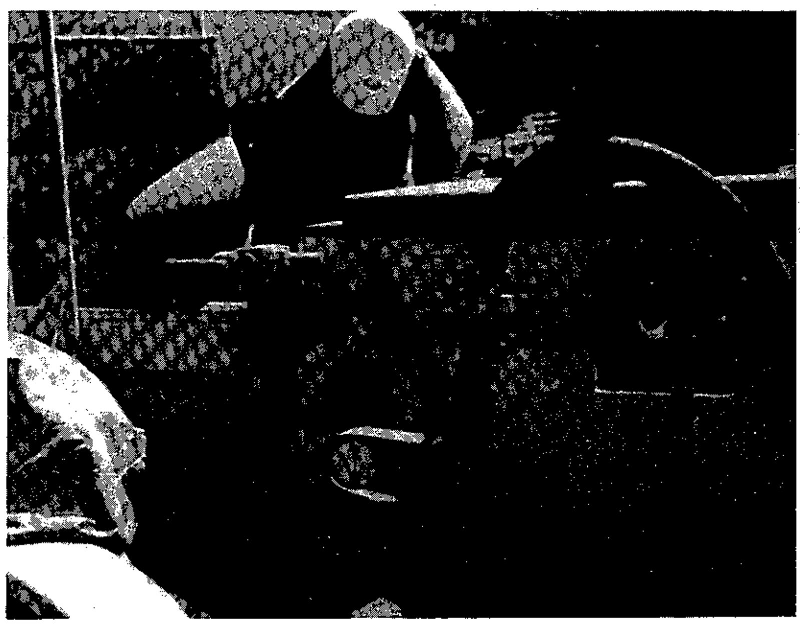


Photo 4: Plaster Mixer with grout delivery system. Note location of pressure gauge.

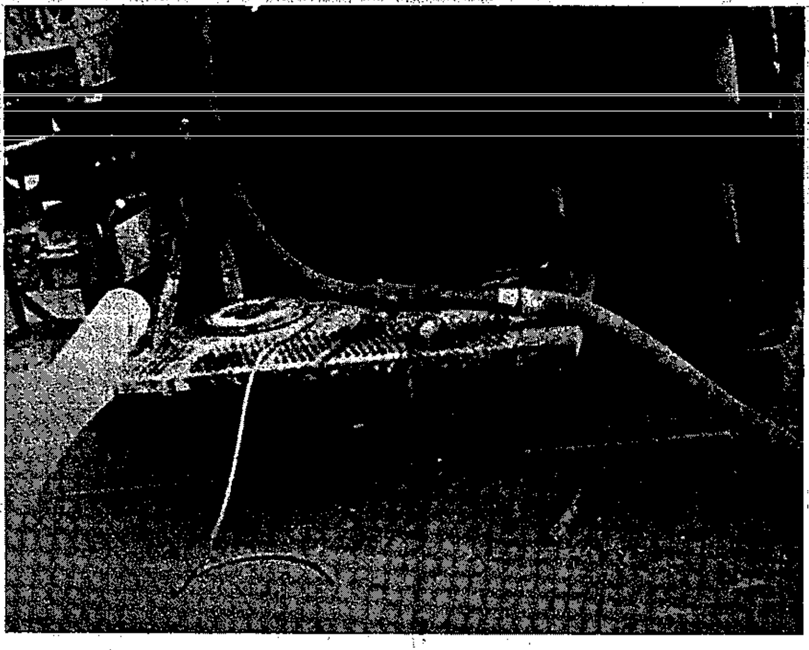


Photo 5: Three-inch diameter grout delivery hose to drill rod stem.

Photo 6: Air Packer for pressure sealing of grout hole with opening at bottom end for grout delivery.

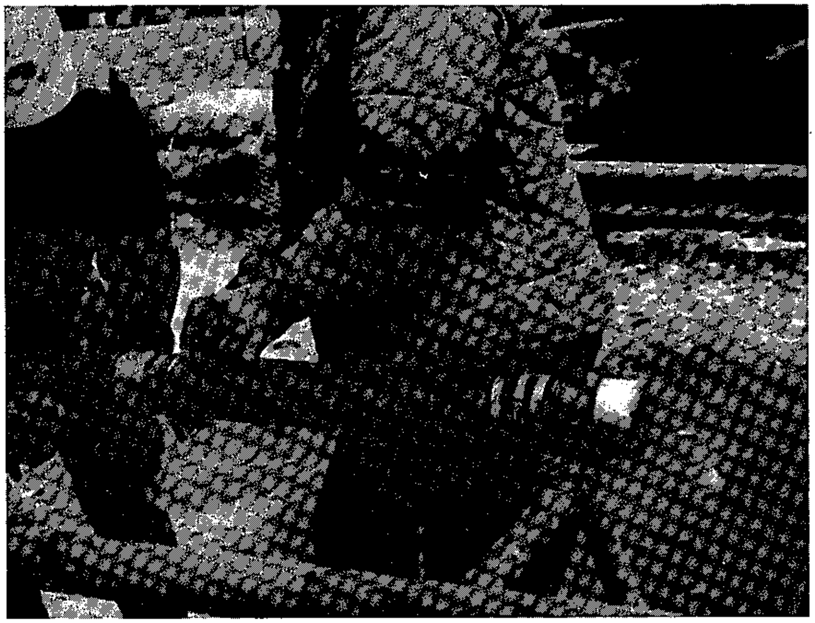


Photo 7: Air Packer being repaired and shortened to 2-foot length.

Photo 8: String line used to monitor pavement heave during grouting. Grout on surface indicates loss of seal above injection point (Packer).